Washington State Department of Transportation

SEG 1B WALLS

I-405; RENTON TO BELLEVUE WIDENING AND EXPRESS TOLL LANES PROJECT

Design Calculations:

WALL CAP CONNECTION CHECK

Prepared By: John Broadus
Checked By: Damon Pellegrini
Reviewed By: Chengyu Li

January 2021

No. _____ Date _____

CHECK PRINT

Dwg. Checked Against Calcs, And Calc. Check Confirmed.

By _____ Date _____

Checked MJF Date 12/22/21
Backchecked _____ Date _____
Corrected _____ Date _____
Verified _____ Date _____

I am just verifying that nothing in this document has changed from the original, QCed design it was apart of. See description on page 3. - MJF

wood.

Environment and Infrastructure Solutions
2000 S. Colorado Blvd, Ste 2-1000
Denver, CO 80222

Table of Contents

WALL CAP CONNECTION CHECK

CALCULATIONS ALREADY SUBMITTED THROUGH REVIEW PROCESS WITH WALL 07.46R -THIS VERSION IS ONLY WITH NEW COVER SO CAN APPLY TO ALL WALLS IN SEGMENT 1B

CALCULATION PACKAGE TO DOCUMENT THE DESIGN CHECK OF THE CONNECTION FOR FALL PROTECTION FENCING TO PRECAST WALL CAP (COPING). THE CALCULATIONS INCLUDE A DESIGN CHECK FOR PEDESTRIAN LOADS IN ACCORDANCE WITH AASHTO LRFD AND FOR FALL PROTECTION LOADS IN ACCORDANCE WITH WAC 296-155-24615. THIS CALCULATION APPLIES FOR CONNECTIONS OF THE WALL CAP ALL WALLS EXCEPT SEW WALL CONNECTIONS, WHICH SHALL BE DESIGNED BY A SUBCONTRACTOR.

CALCULATIONS WERE PERFORMED FOR 6" WALL THICKNESS (MINIMUM WALL THICKNESS ON PROJECT, GOVERNS).

JOB NO.	PS19203160	SHEET	1	OF	4
PHASE	Design	TASK	Wall Cap Connection		
JOB NAME	I-405; Renton To Belleview Widening and Express Toll Lanes Project				
BY	JMB	DATE	7/31/2020		
CHECKED BY	DMP	DATE	8/4/2020		

wood.
Colorado Center Tower II
2000 S. Colorado Blvd., Ste 2-1000
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Concrete Cap Connection Check - FALL PROTECTION ONLY

Input

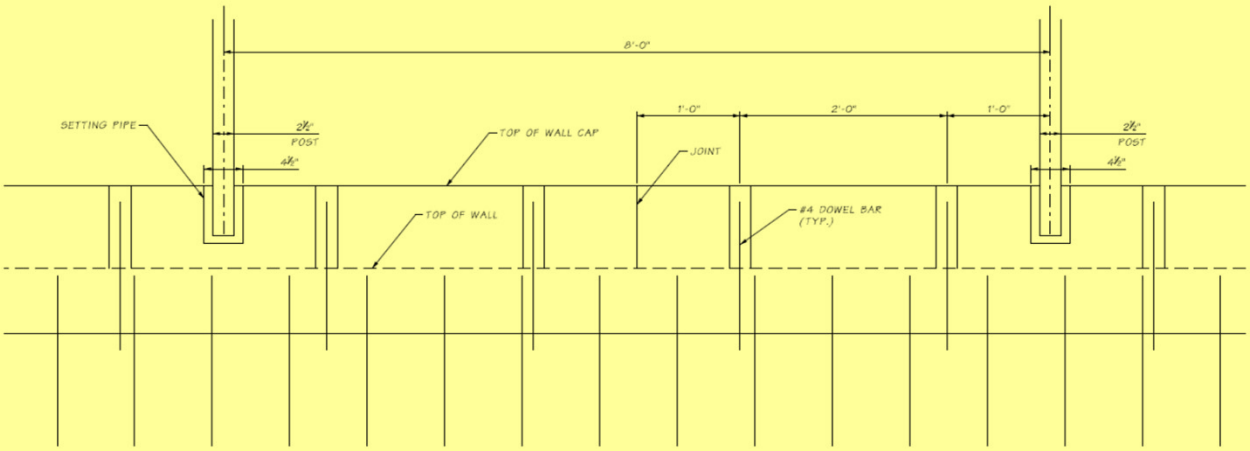
Analysis H = 4.33 ft (from load at top of rail to fixed based in concrete fascia)
stem width = 0.50 ft (note this is minimum/governing expected on project)

Load Factors

design check with Strength I limit state: PL 1.75 LRFD T 3.4.1-1

Fall Protection Load

Loading would transfer into the wall via the wall connection/rebar spaced at an interval to resist the loading between the 8-ft post locations. Assume precast coping to be provided at no more than 8-ft sections per discussion with contractor. Therefore 1 post would act on one coping section. Assume 1 bar on each side of post resists post loading.



P_{LL} = 0.20 k (unfactored) (Per Post)
0.35 k (factored)

Materials Input

f'c = 4.0 ksi BDM 5.1.1.B.2
β1 = 0.85 And per Plan Sheets

fy = 60.0 ksi BDM 5.1.2

γ_e = 0.75 LRFD 5.6.7

modulus of rupture coefficient = 0.24 LRFD 5.4.2.6
fr = 0.480 ksi $0.24 \lambda \sqrt{f'_c}$

wc = 0.155 kcf BDM 3.8

Ec = 4576 ksi $E_c = 120,000 K_1 w_c^{2.0} f_c'^{0.33}$ LRFD 5.4.2.4-1

n = 29000 / 4576 = 6.34

λ = 1.0 LRFD 5.4.2.8

JOB NO.	PS19203160	SHEET	2	OF	4
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Structural Design Loads

Stem Loads

Analysis H = 4.33 ft (height above top of stem where load acts)

Shear

Vu = 0.35 k

Moment

Mu = 1.52 k-ft (Analysis Ht * Vu)
Ms = 0.87 k-ft (as above, unfactored)

Minimum Reinforcement

Minimum Reinforcement LRFD 5.6.3.3
1.33 Mu = 2.02 k-ft

Stem Loading Summary

Vu = 0.35 k
Mu = 2.02 k-ft
Mserv = 0.87 k-ft

Flexural Design

Φ Flexure = 0.9 assumes tension controlled LRFD 5.5.4.2

Stem

b = 48.00 in Tributary Width of Coping Containing Two Connection Bars (1 Ea. Side of Post)
h = 6.00 in
clear cover = 2.75 in
Bar size = 4
0.20 sq in / bar

Coping Embed = 8.00 in Depth Connection is Embedded into Precast Coping
Development Length = 14.40 in Development Length of Connection Bar

$$de = 6.00 - 2.75 - \frac{4}{16} = 3.00 \text{ in}$$

$$dc = 2.75 - \frac{4}{16} = 3.00 \text{ in}$$

# Bars (Legs) = 2
As = 0.222 sq in Efffective Area (Based on Partial Development)

$$\text{stress block depth, } a = \frac{As \times fy}{0.85 \times f'c \times b} = 0.08 \text{ in}$$

Φ Mn = 0.90 x As x fy (de - a/2) = 2.96 k-ft > 2.02 OK

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Crack Control

LRFD 5.6.7

Crack control is required where the tension in the section exceeds 80% modulus of rupture (fr) = 0.480 ksi

80%fr = 0.384 ksi

Compute section area moment to equal criteria above and compare to moment from loading:

$$S = I/c = \frac{=1/12\ (b)\ (h)\text{^}3 =}{c} = \frac{864}{3} = 288\ in^3$$

M*c/I = 0.384

M = 9.216 k-ft > 0.87 OK **Crack control Not Required**

reinforcement ratio $\rho = \frac{As}{b\ de} = \frac{0.222222222}{12.00 \times 3.00} =$ 0.0062

$$k = \sqrt{2n\rho + (n\rho)^2} - n\rho =$$ 0.243

$$j = 1 - k/3 =$$ 0.92

fss = Min (0.6 Fy, $\frac{Mserv\ )}{As\ (j)\ de} =$ 16.98 ksi

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} =$$ 2.43 LRFD 5.6.7-2

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c =$$ 6.73 in LRFD 5.6.7-1

> 2 OK

Shear Design

LRFD 5.7.3.3

Φ Shear = 0.90

β = 2.0

θ = 45° LRFD 5.5.4.2

bv = 12 in LRFD 5.7.3.4.1

Stem

0.9 de = 0.90 x 3.00 = 2.70 in

0.72 h = 0.72 x 6.00 = 4.32 in LRFD 5.7.2.8

dv = max(0.9 de, 0.72 h) = 4.32 in

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}\ b_v d_v$$ LRFD 5.7.3.3-3

Vc = 6.55 k

Φ Vn = 5.90 k

> 0.35 OK

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Shear Friction Interface Check

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c)$$

LRFD 5.7.4.3-4

Ignore c cohesion)
Ignore Pc - net compressive load)

Φ Vn = 0.9*μ*Avf*fy

Avf =	0.2222	in^2	(#4 bar)
fy =	60.0	ksi	
μ =	0.6		
Φ Vn =	7.20	k	
	> 0.35	OK	

LRFD 5.7.4.4

Cohesion and Friction Factors

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened:

c = 0.075 ksi
μ = 0.6
K_1 = 0.2
K_2 = 0.8 ksi

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Concrete Cap Connection Check - FULL PEDESTRIAN LIVE LOAD

Input

Analysis H = 4.33 ft (from load at top of rail to fixed based in concrete fascia)
stem width = 0.50 ft (note this is minimum/governing expected on project)

Load Factors

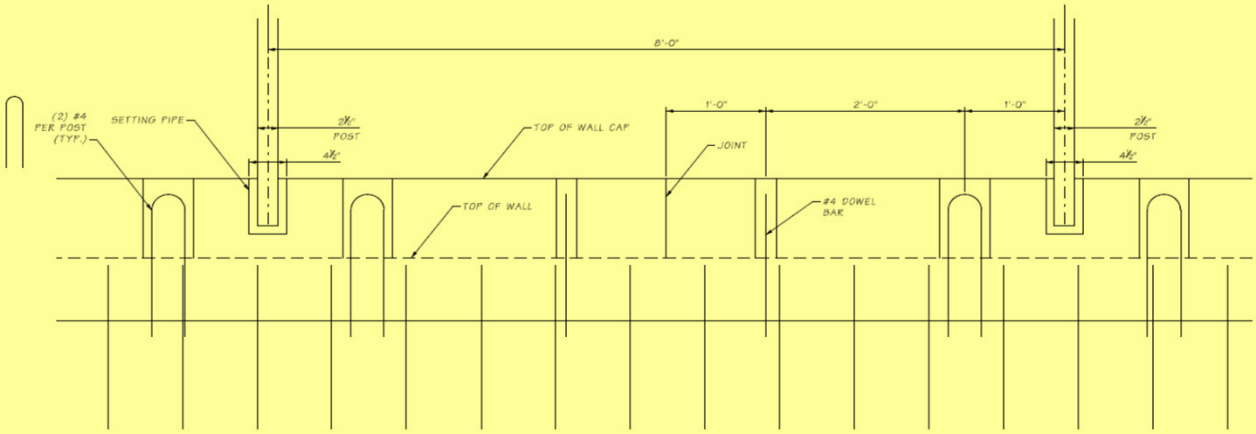
design check with Strength I limit state: PL 1.75 LRFD T 3.4.1-1

Pedestrian Live Load/Fall Restraint Load

$P_{LL} = 0.20 + 0.050L$

L = 8.00 ft
P_{LL} = 0.60 k (unfactored) 1.05 k (factored)

Loading would transfer into the wall via the wall connection/rebar spaced at an interval to resist the loading between the 8-ft post locations. Assume precast coping to be provided at no more than 8-ft sections per discussion with contractor. Therefore 1 post would act on one coping section. Assume hairpin bars on each side of post resist post loading.



P_{LL} = 0.60 k (unfactored) (Per Post)
1.05 k (factored)

Materials Input

f'c = 4.0 ksi — BDM 5.1.1.B.2
β1 = 0.85 — And per Plan Sheets

fy = 60.0 ksi — BDM 5.1.2

γ_e = 0.75 — LRFD 5.6.7

modulus of rupture coefficient = 0.24 — LRFD 5.4.2.6
fr = 0.480 ksi $0.24 \lambda \sqrt{f'_c}$

wc = 0.155 kcf — BDM 3.8

Ec = 4576 ksi $E_c = 120,000 K_1 w_c^{2.0} f_c'^{0.33}$ — LRFD 5.4.2.4-1

n = 29000 / 4576 = 6.34

λ = 1.0 — LRFD 5.4.2.8

JOB NO.	PS19203160	SHEET	2	OF	4
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Structural Design Loads

Stem Loads

Analysis H = 4.33 ft (height above top of stem where load acts)

Shear

Vu = 1.05 k

Moment

Mu = 4.55 k-ft (Analysis Ht * Vu)
Ms = 2.60 k-ft (as above, unfactored)

Minimum Reinforcement

Minimum Reinforcement — LRFD 5.6.3.3

1.33 Mu = 6.05 k-ft

Stem Loading Summary

Vu = 1.05 k
Mu = 6.05 k-ft
Mserv = 2.60 k-ft

Flexural Design

Φ Flexure = 0.9 assumes tension controlled — LRFD 5.5.4.2

Stem

b = 48.00 in Tributary Width of Coping Containing Hairpin Bars
h = 6.00 in
clear cover = 2.75 in
Bar size = 4
0.20 sq in / bar
0.0972

$$de = 6.00 - 2.75 - \frac{4}{16} = 3.00 \text{ in}$$

$$dc = 2.75 - \frac{4}{16} = 3.00 \text{ in}$$

# Bars (Legs) = 4
As = 0.8 sq in

$$\text{stress block depth, } a = \frac{As \times fy}{0.85 \times f'c \times b} = 0.29 \text{ in}$$

Φ Mn = 0.90 x As x fy (de - a/2) = 10.27 k-ft > 6.05 OK

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Crack Control

LRFD 5.6.7

Crack control is required where the tension in the section exceeds 80% modulus of rupture (fr) = 0.480 ksi

80%fr = 0.384 ksi

Compute section area moment to equal criteria above and compare to moment from loading:

$$S = I/c = \frac{=1/12\ (b)\ (h)\text{^}3 =}{c} = \frac{864}{3} = 288\ in^3$$

M*c/I = 0.384

M = 9.216 k-ft > 2.60 OK **Crack control Not Required**

reinforcement ratio $\rho = \frac{As =}{b\ de} = \frac{0.8}{12.00 \times 3.00} = 0.0222$

$$k = \sqrt{2n\rho + (n\rho)^2} - n\rho = 0.408$$

$$j = 1 - k/3 = 0.86$$

fss = Min (0.6 Fy, $\frac{Mserv\ )}{As\ (j)\ de} =$ 15.05 ksi

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} = 2.43$$ LRFD 5.6.7-2

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c = 8.37\ in$$ LRFD 5.6.7-1

> 4 OK

Shear Design

LRFD 5.7.3.3

Φ Shear = 0.90

β = 2.0

θ = 45° LRFD 5.5.4.2

bv = 12 in LRFD 5.7.3.4.1

Stem

0.9 de = 0.90 x 3.00 = 2.70 in

0.72 h = 0.72 x 6.00 = 4.32 in LRFD 5.7.2.8

dv = max(0.9 de, 0.72 h) = 4.32 in

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}\ b_v d_v$$ LRFD 5.7.3.3-3

Vc = 6.55 k

Φ Vn = 5.90 k

> 1.05 OK

JOB NO.	PS19203160	SHEET	4	OF	4
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Shear Friction Interface Check

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c)$$

LRFD 5.7.4.3-4

Ignore c cohesion)
Ignore Pc - net compressive load)

Φ Vn = 0.9*μ*Avf*fy

Avf =	0.8	in^2	(#4 bar)
fy =	60.0	ksi	
μ =	0.6		
Φ Vn =	25.92	k	
	> 1.05	OK	

LRFD 5.7.4.4

Cohesion and Friction Factors

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened:

c = 0.075 ksi
μ = 0.6
K_1 = 0.2
K_2 = 0.8 ksi

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Concrete Cap Connection Check - FULL PEDESTRIAN LIVE LOAD @ WALL STEP

Input

Analysis H1 =	5.83	ft	(from load at top of high step rail to fixed based in concrete fascia)
Analysis H2 =	4.33	ft	(from load at top of low step rail to fixed based in concrete fascia)
stem width =	0.50	ft	(note this is minimum/governing expected on project)

Load Factors

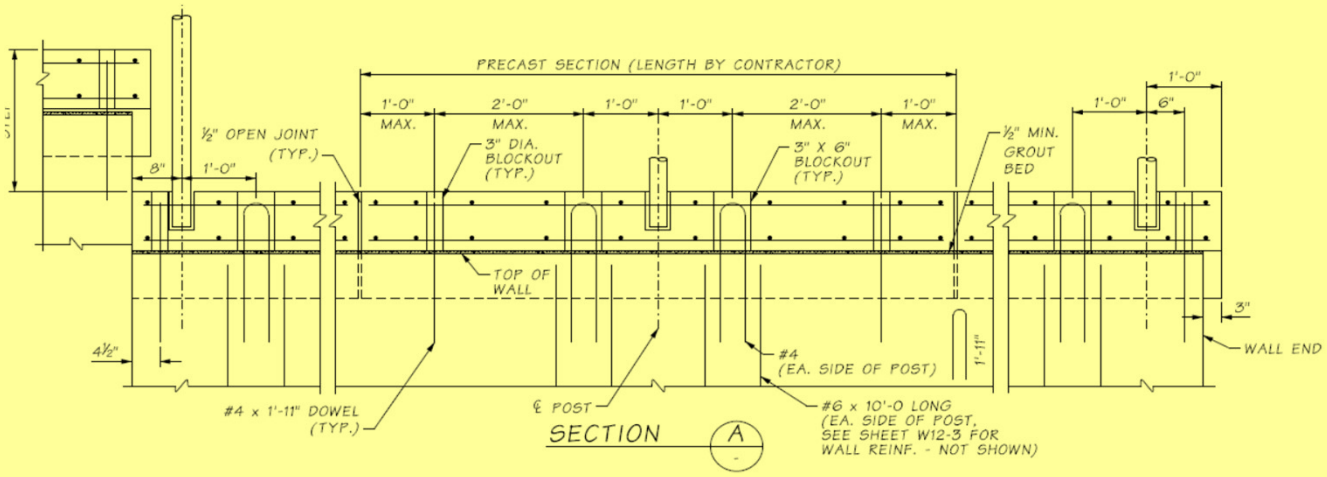
design check with Strength I limit state: PL 1.75 LRFD T 3.4.1-1

Pedestrian Live Load/Fall Restraint Load

P_{LL1} = 0.20 + 0.050(L1/2) P_{LL2} = 0.050(L2/2)

L1 =	4.00	ft (High Step)	
L2 =	8.00	ft (Low Step)	
P_{LL1} =	0.30	k (unfactored)	0.525 k (factored)
P_{LL2} =	0.20	k (unfactored)	0.35 k (factored)

Loading would transfer into the wall via the wall connection/rebar spaced at an interval to resist the loading between the 8-ft post locations. Assume precast coping to be provided at no more than 8-ft sections per discussion with contractor. Therefore 1 post would act on one coping section. Use one hairpin bar on the inside of post spacing and partially developed single dowel to resist post loading.



P_{LL} = 0.50 k (unfactored) (Per Post)
0.88 k (factored)

Materials Input

f'c =	4.0	ksi		BDM 5.1.1.B.2
β1 =	0.85			And per Plan Sheets
fy =	60.0	ksi		BDM 5.1.2
γ_e =	0.75			LRFD 5.6.7
modulus of rupture coefficient =	0.24			LRFD 5.4.2.6
fr =	0.480	ksi	$0.24\,\lambda\sqrt{f'_c}$	
wc =	0.155	kcf		BDM 3.8
Ec =	4576	ksi	$E_c = 120{,}000 K_1 w_c^{2.0} f_c'^{0.33}$	LRFD 5.4.2.4-1
n = 29000 / 4576 =	6.34			
λ =	1.0			LRFD 5.4.2.8

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Structural Design Loads

Stem Loads

Analysis H1 = 5.83 ft (height above top of stem where load acts)
Analysis H2 = 4.33 ft (height above top of stem where load acts)

Shear

Vu = 0.88 k

Moment

Mu = 4.58 k-ft Analysis H1*P_{LL1}*LF + Analysis H2*PLL2*LF
Ms = 2.62 k-ft (as above, unfactored)

Minimum Reinforcement

Minimum Reinforcement LRFD 5.6.3.3
1.33 Mu = 6.09 k-ft

Stem Loading Summary

Vu = 0.88 k
Mu = 6.09 k-ft
Mserv = 2.62 k-ft

Flexural Design

Φ Flexure = 0.9 assumes tension controlled LRFD 5.5.4.2

Stem

b = 48.00 in Tributary Width of Coping Containing Hairpin Bars
h = 6.00 in
clear cover = 2.75 in
Bar size = 4 Hairpin Bar
0.20 sq in / bar
Bars (Legs) = 2 Hairpin Bar

Bar size = 4 Single Dowel
0.20 sq in / bar
Bars (Legs) = 1 Single Dowel

Coping Embed = 8.00 in Depth Single Dowel is Embedded into Precast Coping
Development Length = 14.40 in Development Length of Single Dowel Connection Bar

As = 0.51 sq in

$$de = 6.00 - 2.75 - \frac{4}{16} = 3.00 \text{ in}$$

$$dc = 2.75 - \frac{4}{16} = 3.00 \text{ in}$$

$$\text{stress block depth, } a = \frac{As \times fy}{0.85 \times f'c \times b} = 0.19 \text{ in}$$

Φ Mn = 0.90 x As x fy (de - a/2) = 6.68 k-ft > 6.09 OK

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Crack Control

LRFD 5.6.7

Crack control is required where the tension in the section exceeds 80% modulus of rupture (fr) = 0.480 ksi

80%fr = 0.384 ksi

Compute section area moment to equal criteria above and compare to moment from loading:

$$S = I/c = \frac{=1/12\ (b)\ (h)\text{^}3 =}{c} = \frac{864}{3} = 288\ in^3$$

M*c/I = 0.384

M = 9.216 k-ft > 2.62 OK Crack control Not Required

reinforcement ratio $\rho = \frac{As =}{b\ de} = \frac{0.511111111}{12.00 \times 3.00} =$ 0.0142

$$k = \sqrt{2n\rho + (n\rho)^2} - n\rho =$$ 0.344

$$j = 1 - k/3 =$$ 0.89

fss = Min (0.6 Fy, $\frac{Mserv\ )}{As\ (j)\ de} =$ 23.13 ksi

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} =$$ 2.43 LRFD 5.6.7-2

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c =$$ 3.35 in LRFD 5.6.7-1

> 2 OK

Shear Design

LRFD 5.7.3.3

Φ Shear = 0.90

β = 2.0

θ = 45° LRFD 5.5.4.2

bv = 12 in LRFD 5.7.3.4.1

Stem

0.9 de = 0.90 x 3.00 = 2.70 in

0.72 h = 0.72 x 6.00 = 4.32 in LRFD 5.7.2.8

dv = max(0.9 de, 0.72 h) = 4.32 in

$$V_c = 0.0316\,\beta\,\lambda\sqrt{f'_c}\ b_v\,d_v$$ LRFD 5.7.3.3-3

Vc = 6.55 k

Φ Vn = 5.90 k

> 0.88 OK

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Shear Friction Interface Check

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c)$$

LRFD 5.7.4.3-4

Ignore c cohesion)
Ignore Pc - net compressive load)

Φ Vn = 0.9*μ*Avf*fy

Avf =	0.5111	in^2	(#4 bar)
fy =	60.0	ksi	
μ =	0.6		
Φ Vn =	16.56	k	
	> 0.88	OK	

LRFD 5.7.4.4

Cohesion and Friction Factors

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened:

c = 0.075 ksi
μ = 0.6
K_1 = 0.2
K_2 = 0.8 ksi

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Concrete Cap Connection Check - FULL PEDESTRIAN LIVE LOAD @ WALL END

Input

Analysis H = 4.33 ft (from load at top of rail to fixed based in concrete fascia)
stem width = 0.50 ft (note this is minimum/governing expected on project)

Load Factors

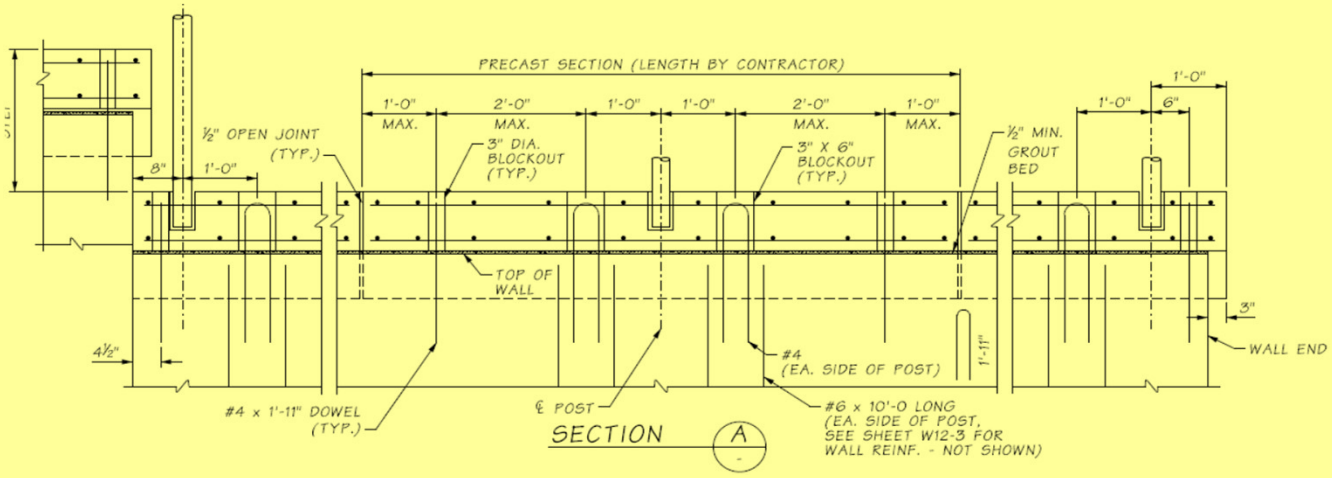
design check with Strength I limit state: PL 1.75 LRFD T 3.4.1-1

Pedestrian Live Load/Fall Restraint Load

$P_{LL} = 0.20 + 0.050L$

L = 4.00 ft (1/2 th post space)
P_{LL} = 0.40 k (unfactored) 0.7 k (factored)

Loading would transfer into the wall via the wall connection/rebar spaced at an interval to resist the loading between the 8-ft post locations. Assume precast coping to be provided at no more than 8-ft sections per discussion with contractor. Therefore 1 post would act on one coping section. Conservatively assume the one hairpin bar on the inside of post spacing resists post loading and neglect the dowel bar at the very end.



P_{LL} = 0.40 k (unfactored) (Per Post)
0.70 k (factored)

Materials Input

f'c = 4.0 ksi BDM 5.1.1.B.2
β1 = 0.85 And per Plan Sheets

fy = 60.0 ksi BDM 5.1.2

γ_e = 0.75 LRFD 5.6.7

modulus of rupture coefficient = 0.24 LRFD 5.4.2.6
fr = 0.480 ksi $0.24 \lambda \sqrt{f'_c}$

wc = 0.155 kcf BDM 3.8

Ec = 4576 ksi $E_c = 120,000 K_1 w_c^{2.0} f_c'^{0.33}$ LRFD 5.4.2.4-1

$n = \frac{29000}{4576} = 6.34$

λ = 1.0 LRFD 5.4.2.8

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PHASE	Design	TASK	Wall Cap Connection		
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BY	JMB	DATE	7/31/2020		
CHECKED BY	DMP	DATE	8/28/2020		

wood.
Colorado Center Tower II
2000 S. Colorado Blvd., Ste 2-1000
Denver, CO 80222
+1 (303) 935-6505 Fax '+1 (303) 935-6575

Structural Design Loads

Stem Loads

Analysis H = 4.33 ft (height above top of stem where load acts)

Shear

Vu = 0.70 k

Moment

Mu = 3.03 k-ft (Analysis Ht * Vu)
Ms = 1.73 k-ft (as above, unfactored)

Minimum Reinforcement

Minimum Reinforcement LRFD 5.6.3.3

1.33 Mu = 4.03 k-ft

Stem Loading Summary

Vu = 0.70 k
Mu = 4.03 k-ft
Mserv = 1.73 k-ft

Flexural Design

Φ Flexure = 0.9 assumes tension controlled LRFD 5.5.4.2

Stem

b = 48.00 in Tributary Width of Coping Containing Hairpin Bars
h = 6.00 in
clear cover = 2.75 in
Bar size = 4
0.20 sq in / bar
0.0972

$$de = 6.00 - 2.75 - \frac{4}{16} = 3.00 \text{ in}$$

$$dc = 2.75 - \frac{4}{16} = 3.00 \text{ in}$$

# Bars (Legs) = 2
As = 0.4 sq in

$$\text{stress block depth, } a = \frac{As \times fy}{0.85 \times f'c \times b} = 0.15 \text{ in}$$

Φ Mn = 0.90 x As x fy (de - a/2) = 5.27 k-ft > 4.03 OK

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Crack Control

LRFD 5.6.7

Crack control is required where the tension in the section exceeds 80% modulus of rupture (fr) = 0.480 ksi

80%fr = 0.384 ksi

Compute section area moment to equal criteria above and compare to moment from loading:

$$S = I/c = \frac{=1/12\ (b)\ (h)\text{^}3 =}{c} = \frac{864}{3} = 288\ in^3$$

M*c/I = 0.384

M = 9.216 k-ft > 1.73 OK **Crack control Not Required**

reinforcement ratio $\rho = \frac{As =}{b\ de} = \frac{0.4}{12.00 \times 3.00} =$ 0.0111

$$k = \sqrt{2n\rho + (n\rho)^2} - n\rho =$$ 0.311

$$j = 1 - k/3 =$$ 0.90

fss = Min (0.6 Fy, $\frac{Mserv\ )}{As\ (j)\ de} =$ 19.34 ksi

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} =$$ 2.43 LRFD 5.6.7-2

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c =$$ 5.18 in LRFD 5.6.7-1

> 2 OK

Shear Design

LRFD 5.7.3.3

Φ Shear = 0.90

β = 2.0

θ = 45° LRFD 5.5.4.2

bv = 12 in LRFD 5.7.3.4.1

Stem

0.9 de = 0.90 x 3.00 = 2.70 in

0.72 h = 0.72 x 6.00 = 4.32 in LRFD 5.7.2.8

dv = max(0.9 de, 0.72 h) = 4.32 in

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}\ b_v d_v$$ LRFD 5.7.3.3-3

Vc = 6.55 k

Φ Vn = 5.90 k

> 0.70 OK

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Shear Friction Interface Check

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c)$$

LRFD 5.7.4.3-4

Ignore c cohesion)
Ignore Pc - net compressive load)

Φ Vn = 0.9*μ*Avf*fy

Avf =	0.4	in^2	(#4 bar)
fy =	60.0	ksi	
μ =	0.6		
Φ Vn =	12.96	k	
	> 0.70	OK	

LRFD 5.7.4.4

Cohesion and Friction Factors

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened:

c = 0.075 ksi
μ = 0.6
K_1 = 0.2
K_2 = 0.8 ksi